Appendix C

Geotechnical Design Requirements

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1.0 Introduction

This document presents the preliminary geotechnical evaluation of the proposed Bel Marin Keys V (BMK-V) expansion to the authorized Hamilton Wetland Restoration Project (HWRP). Relevant information on the regional geological setting and geotechnical properties of the site is provided here.

The BMK-V parcel is directly adjacent to the Hamilton Airfield, to the north, on the fringe of San Pablo Bay. Both properties historically consisted of nearly level mudflats and tidal marsh that was levied off from tidal action to create agricultural lands. The first levees around the BMK site were constructed in 1892. Both parcels of land are underlain by thick layers of Bay Mud and have settled significantly as a result of the separation of waters of San Pablo Bay and the surrounding watershed. The majority of the geotechnical data used in this evaluation were obtained from the 1995 BMK site investigation conducted by Miller Pacific Engineering Group for the California Quartet's proposed residential development of the property. However, due to the close proximity, geomorphic continuity, and similar recent history of the two sites, valuable qualitative information for BMK may be derived from existing investigations of the Hamilton Airfield property as well.

The US Army Corps of Engineers have identified four primary geotechnical issues that potentially impact the proposed project expansion. These include long-term settlement, compactability of dredged material, levee stability, and levee performance during and after an earthquake.

For the purposes of this evaluation, it is assumed that the subsurface site conditions do not deviate significantly from those described in the subject report[1]. The basic design and construction procedures for levees and embankments provided in the subject report are very similar to those proposed for the BMK expansion to the HWRP. The estimates provided here would be refined based on additional site-specific information obtained during the pre-construction, engineering and design phase of the expanded project.

<u>Previous Studies</u>

The following documents were used as reference material for the analysis contained in this appendix.

1. Geotechnical Investigation Bel Marin Keys Unit 5 Marin County, California, Volumes I and II, December 21, 1995, Miller Pacific Engineering Group
2. Draft Supplemental Environmental Impact Report/Environmental Impact Statement (SEIR/EIS), Bel Marin Keys Unit V Expansion of the Hamilton Wetland Restoration Project, May 2002, Jones & Stokes.

[1] Geotechnical Investigation Bel Marin Keys Unit 5 Marin County, California, Volumes I and II, December 21, 1995, Miller Pacific Engineering Group

3. Hamilton Restoration Plan, Volume I: Feasibility Study. December 1998. California State Coastal Conservancy and the U.S. Army Corps of Engineers, San Francisco District.

2. Regional Geologic Setting

The site is located within California's geologically and seismically active Coast Range Geomorphic Province. The province is characterized by a series of northwest trending faults, mountain ranges, and valleys. These include three dominant faults: the Hayward fault to the southeast, the San Andreas fault to the west, and the Healdsburg-Rogers Creek fault to the northeast (Environmental Science Associates 1993 as cited in the Draft SEIR/EIS May 2002). The Draft Supplemental Environmental Impact Report/Environmental Impact Statement (SEIR/EIS) for Bel Marin Keys Unit V Expansion of the Hamilton Wetland Restoration Project (HWRP) describes in detail factors that could potentially impact the project site. These include, potential surface fault rupture, ground failure hazards, seismicity and geologic hazards, ground shaking, and earthquake-induced inundation.

However, no active or potentially active faults are known to exist within the boundaries of the site. In addition, the site is not within the Alquist-Priolo Special Studies Zone, as designated by the California Division of Mines and Geology (Hart and Bryant 1997 as cited in the Draft SEIR/EIS May 2002). Accordingly, the potential for surface fault rupture to occur at the site is remote (Miller Pacific Engineering Group 1995).

The site is underlain by unconsolidated alluvium. Ground shaking at the site during an earthquake is likely to be more intense than in nearby areas underlain by bedrock. The magnitude of the maximum credible earthquake is: 7.5 Richter scale magnitude (M) for Hayward fault, 8.3 M for San Andreas faults, 7.2 M for the Healdsburg-Rogers Creel fault (Draft SEIR/EIS May 2002). Although ground shaking may be intense, there is little or no potential for seismic settlement to occur because the soils and bay mud that underlie the site consist of clays and silty clays rather than clean sands and silts (Miller Pacific Engineering Group 1995). However, cracking of the ground surface ('lurch cracking') during an earthquake may occur along the edge of embankments underlain by soft, compressible soils (Miller Pacific Engineering Group 1995).

2.1 Geotechnical Conditions

As mentioned above, both the BMK and Hamilton sites have subsided from historic marsh plain levels and are currently below sea level. Current elevations for the BMK parcel range from approximately –4 feet to –5 feet on the National Geodetic Vertical Datum (NGVD, 1929). The site is protected from tidal inundation by flood control levees along San Pablo Bay and Novato Creek, and by a system of drainage trenches and pumps.

According to the Soil Survey of Marin County (Kashiwagi 1985 as cited in Draft SEIR/EIS May 2002), the bay mud deposits that underlie the site are overlain entirely by soils of the Reyes series, also referred to as "desiccated crust." Desiccated crust is the upper, near surface, layer of the Bay Mud exposed for a sufficiently long time for the loss of moisture to cause over consolidation, shrinkage and oxidation of the soil, i.e. mottling.

Soils of the Reyes series typically consist of slowly permeable clays and silty clays (Miller Pacific Engineering Group (1995). Soils of the Reyes series are still susceptible to settlement when dewatered or subjected to large static fill loads. Due to the fine texture of the Reyes soil and the low slope gradients that prevail at the site, the hazard of soil erosion is slight.

The bay mud is a plastic, silty clay to clayey silt, with high compressibility, low shear strength, and generally low permeability. Compressibility properties for the Reyes clay soils and bay mud are shown in Table 1.

Table 1. Compressibility properties for bay mud and Reyes soils at BMK (Miller Pacific Engineering Group 1995).

Compressibility property	Reyes soil (“desiccated crust”)	Bay mud
Virgin compression ratio, $C_{\varepsilon C}$	0.21	0.36
Vertical coefficient of consolidation, C_V (ft^2/yr)	10	10

The bay mud is underlain by much stronger and less compressible alluvial and residual soils and bedrock (Miller Pacific Engineering Group, 1995) Due primarily to its high compressibility and low strength, the soft bay mud poses considerable challenges to development of the site as a wetland. New fill loads cause compression of underlying bay mud, and may cause uneven settlement of the surface. Depending on the depth of the underlying bay mud, the settlement may continue for long periods of up to 50 to 100 years.

Settlement curves for estimating the anticipated amount of settlement of soft bay mud were generated for a proposed residential development by Miller Pacific Engineering Group in 1995. In the adjacent HWRP project, conceptual settlement curves were also generated for large-area fill loads (e.g. general blanket fills) and more localized fill loads (e.g. levee fills). Fills applied over limited areas may cause shear stresses in the bay mud that, if they exceed the soil’s shear strength, could cause stability failures of the levees.

3. Geotechnical Issues Common to BMK Expansion Alternatives 1, 2, and 3

The three alternatives under consideration for this study are:

1) Dredged material placement with an enlarged Pacheco Pond
2) Dredged material placement with seasonal wetlands
3) Natural sedimentation with an enlarged Pacheco Pond.

The key geotechnical characteristics common to the three alternatives are described below.

3.1 Levees and Tidal Wetland Settlement

The amount of settlement will vary depending on the thickness of the fill placed, groundwater level and the thickness of underlying Bay Mud (Miller Pacific Engineering Group, 1995). Bay Mud thickness varies from 30 feet on the west side of the site near Pacheco Pond to 90 feet on the northeast corner bordering Novato Creek. Because of the varying thickness of bay mud thickness and levee/tidal marsh, it is likely that differential settlements will occur. Under Alternatives 1-3, differential settlement is likely to be encountered on the northwestern portion of the site where abrupt changes in the thickness of the Bay Mud deposits are reported and near the expanded portion of Pacheco Pond under Alternatives 1 and 3.

For this preliminary evaluation, settlement estimates were done using soil consolidation data from reports of Miller Pacific Engineering (1995) and the HWRP Feasibility Study (1998). Settlement curves from HWRP Feasibility Study cover a 50-year design period. On the other hand, a 100-year design period was used to generate the settlement curves from Miller Pacific report. The settlement curves from Miller Pacific report were developed from laboratory consolidation tests performed on bay mud samples obtained at the BMKV site.

3.1.1 Tidal/Non-Tidal Marsh Settlement

The amount of settlement associated with placement of dredged material in the tidal and non-tidal marsh areas was estimated using compressibility data presented in Table 1 and the conceptual settlement curves provided in HWRP Feasibility Study report. Settlement estimates in the marsh areas are presented in Table 2. The total settlement of foundation soil in the tidal marsh area is expected to vary from less than 1 foot to greater than 3 feet depending on the bay mud thickness. The estimated settlement values presented in Table 2 are long-term settlement that may vary from 30 years for a 30 feet thick bay mud to more than 50 years for a 90 feet thick bay mud. For the first 5 years after dredged material placement, the settlement values are estimated to range from 0.3 to 1.8 feet depending on fill thickness. In general, the expected settlement using the Miller Pacific data appears to produce greater settlement compared with the conceptual settlement estimates using the HWRP curves (Table 2).

Table 2. Estimated Settlement and Dredged Fill Material Requirements for Tidal Marsh Areas of the Proposed BMK-V Expansion.#

Geotechnical Feature	Approx. Present Ground surface Elevation (ft) NGVD	Estimated Thickness of bay mud (ft)	Ground-water Elevation (ft)	Top Elevation of feature (ft)	Approx. Height of Feature (ft)	Fill Thickness Needed *	Estimated total settlement in feet using Miller Pacific soils data (1995)	Estimated total settlement in feet using HWRP curves** (1998)
Tidal marsh	-5	20	-5	2	7	7.8	NA	0.8
	-5	40	-5	2	7	(8.5-9.9)	2.9	1.5
	-5	60	-5	2	7	(9.0-10.2)	3.2	2.0

* Fill thickness required to meet final design elevation for one time placement
** HWRP settlement curves available only for 20, 40 and 60 feet Bay Mud
(8.5-9.9) - fill thickness needed using both settlement estimates from HWRP & Miller, respectively
\# Total settlement estimates based on 30 to more than 50 years (HWRP); and 100 years (Miller Pacific)
Top elevation obtained from draft SEIR/EIS May 2002 report.

3.1.2 Levee Settlement

Settlements for new and improved levees proposed in the BMK expansion alternatives will vary depending on the thickness of the fill placed, groundwater level and the thickness of underlying Bay Mud. For both levee construction and tidal marsh restoration, the expected settlement of the underlying Bay Mud due to the weight of the applied fill material between 10 and 30 percent of total fill height.

As the site is underlain by Bay Mud that is highly compressible, the design must account for settlement of the levees that will occur over time. An alternative design measure, staged construction, was evaluated to address the settlement issue. If staged construction were employed, levee material would have to be temporarily stored on site for future use in levee staged construction, as the restored habitat would cover the borrow sites.

Staged construction merits further consideration as this would address the visual impact issue important to the BMK residential community. To reduce any visual impact resulting from levee height construction near the existing BMK residential community, fill placement for new levees may be done in two stages or more over the design period of 50 years. A concept similar to the method developed by Olson (1977) and later adapted in NAVFAC DM7.1 was used to estimate the consolidation settlement under time-dependent loading.

Fill placement will consist of constructing the flood control levee in two or more stages to an initial elevation of 10.0 feet NGVD or 15 feet of fill. Settlement is expected after each fill placement. It is estimated that in 50 years the crest elevation will stabilize to a crest elevation of about 8.0 feet.

The improved levee separating the BMK south lagoon from the project site has an existing crest elevation from 2 to 5 feet NGVD. It is estimated that in more than 10 years since the south lagoon levee was constructed about 60 percent consolidation has occurred leaving approximately one foot of residual settlement. This calculation is based on consolidation of 40-foot thick San Francisco Bay Mud, which appear to underlie the improved levee (Miller Pacific Engineering, 1995). The total settlement of 2.5 feet was obtained using the conceptual settlement curves from the adjacent Hamilton Airfield Wetland Restoration Project.

The proposed improvements to the south lagoon levee will consist of fill placement on the levee to elevation 6.0 feet and about 2 to 3 feet of fill at the toe of the levee on the wetland side. It is estimated that the additional fill can be accommodated to achieve a final crest elevation of 5 feet NGVD.

The estimates on fill thicknesses, settlements, and fill staging sequence must be verified by conducting a subsurface sampling and testing program at the subject site and closely monitored during construction to avoid over or underfilling.

3.2 Dredged Material Properties for Fill Material and Levee Construction

The levees and tidal marsh area will be constructed using fill material excavated from the site and possibly dredged material imported from offsite sources such as the Port of Oakland, Richmond Harbor, Pinole Shoal Channel, and Petaluma River Channel. Table 3 provides information on quantities and material types from offsite sources of dredged material for use at the project site.

Table 3. Dredged material type distribution (Moffat & Nichols Engineers).

Material Type	Quantity (cu.yd.)	% Total
Dense sand	2,000,000	19.64
Sandy silt	794,000	7.8
Silty loam	1,060,680	10.41
Silt & clays	6,330,020	62.15

Alternatives 1 and 2 would utilize dredged material to establish initial surface elevations of the wetlands and potentially to create levees if adequate on site borrow material was not readily available. Initially, the dredged material that is pumped to the site is expected to contain 20% dredged material and 80% water by volume, which means that the dredged material may be too wet and not suitable for compaction. In order for the dredged material to be used as fill material for levee construction, dewatering may need to be done in a location off the fill areas. This would allow the dredged material to dry and be moisture-conditioned to make it suitable for compaction. Issues related placement and compaction of the material need to be addressed during the design and construction phases. Since about 25% of the material consists of sand, soil stabilization may be required if this material is used for levee construction.

Prior to the use of dredged material for levee construction, testing and analysis must be carried out to confirm its suitability for use. The laboratory testing should include the determination of the material's compaction characteristics; shear strength of the compacted material, compressibility, and expansion potential for use in the design. A literature search of the engineering characteristics of the dredged material should also be conducted to provide supplemental information for the design phase.

3.3 Levee Stability

Slopes of levees constructed with onsite soils (not dredged material) and having an inclination of 2H:1V to 3H:1V (horizontal to vertical) generally have been shown by Miller Pacific Engineering Group to have factors of safety greater than the recommended FS of 1.3 (USACE EM 1110-2-1913). Existing levee top elevations at different locations are summarized in Table 4. Stability analyses will be performed for the levees using appropriate shear strength parameters obtained from laboratory testing. A computer program such as UTEXAS3 developed by the University of Texas or other equivalent programs will be used to perform the analyses. Similarly, the factors of safety against bearing capacity failure must also be analyzed to ensure that the bay mud shear strengths will not be exceeded.

Table 4. Summary of Existing Levee Elevations Surrounding the BMK-V Parcel

Levee boundary location	Approximate levee top elevation (NGVD)
BMK-V / San Pablo Bay	6-10
BMK-V / Novato Creek	5-8
BMK-V / BMK lagoon	2-5
BMK-V / Pacheco Pond	8-11
BMK-V / Hamilton Airfield	1-5

Ref: Draft SEIR/EIS for BMKV, May 2002, Jones & Stokes.

3.4 Seismic Considerations

Ground failure hazards such as liquefaction, earthquake-induced settlement, and lurching are processes that involve the displacement of the ground surface resulting from a loss of strength or failure of the underlying materials due to strong seismic ground motions.

Because the Reyes soils (‘desiccated crust’) and bay mud deposits do not contain substantial quantities of clean sands and silts, they are not conducive to liquefaction (the sudden loss of soil strength during strong ground shaking) and earthquake-induced settlement. However, there is a potential for earthquake-induced lurch cracking to occur at the site during an earthquake (Miller Pacific Engineering Group 1995).

3.5 Novato Sanitation District Outfall

The new installation of a new sanitary outfall pipeline would be located slightly below the grade of the existing pipeline and along the alignment that separates the project site from the adjacent HWRP parcel and around the east side of the expanded Pacheco Pond. As described in more fully in the BMK-V EIR/EIS, the existing sewer outfall pipeline would be replaced because of potential differential settling and leakage resulting from levee construction and tidal marsh restoration.

3.6 Utility Protection

To protect the 5 electric transmission line towers on the proposed BMK expansion area from erosion and corrosion, concrete jackets will be constructed at the base of the towers. Utility service will not be affected during this activity. Experience from the Sonoma Baylands Wetland Restoration Project, which has similar towers for the same power transmission lines that also cross that site, indicates that there are minimal impacts to the foundations of the towers associated with placement of dredged material around the tower base. However, should future work reveals potential impact on tower foundation capacity as a result of down drag forces from weight of overlying fill and settlement of the underlying bay mud, necessary investigative and corrective measures should be done.

Geotechnical Issues Unique to Alternatives

♦ **Alternative 1**: Dredged Material Placement with Enlarged Pacheco Pond

Alternative 1 would entail enlarging the existing Pacheco Pond by approximately 50 acres, which would result in greater fresh water habitat. Under this alternative, the bottom elevation of Pacheco Pond would remain at the existing elevation of –3 feet NGVD and the water surface level at +1.5 feet NGVD. A new flood protection levee would be constructed along the east side of Pacheco Pond and would extend north to Novato Creek. Potential differential settlement would likely occur at the transition from the levee to Pacheco Pond because of varying overburden pressure and abrupt change in bay mud thickness in the area.

♦ **Alternative 2**: Dredged Material Placement with Seasonal Wetlands

Alternative 2 would expand the existing Pacheco Pond and the swale south of the BMK south lagoon. This would involve constructing improved and new levees to lower design heights of 5 feet and 8 feet NGVD, respectively. A levee with an initial top elevation of approximately 10 feet NGVD (with a 2-foot settlement allowance, resulting in a design elevation of 8 feet NGVD) would be constructed across the middle portion of the site to separate the non-tidal and tidal habitats. The existing levee along the BMK south lagoon would be improved to an initial top elevation of 6 feet NGVD, which includes a 1-foot settlement allowance, resulting in a design elevation of 5 feet NGVD. Staged fill placement for new levee construction would be done in two stages or more over the design period of 50 years. Settlement is expected after each fill placement but is estimated to stabilize to the design crest elevation.

♦ **Alternative 3**: Natural Sedimentation with Enlarged Pacheco Pond

With the exception of levee construction and dredge material placement in the tidal/non-tidal marsh basins, Alternative 3 is identical to Alternative 1 in terms of the creation of an enlarged Pacheco Pond. Approximately 727,000 cubic yards of dredged material would be required to create levees, interior peninsulas and berms.

5. Summary and Recommendations

To address the various geotechnical concerns associated with the Bel Marin Keys V expansion, detailed geotechnical site investigations need to be conducted to generate data specific to the site and project features. The investigations would further evaluate subsurface conditions encountered at the site (e.g. thickness and compressibility of the bay mud deposits).

5.1 Settlement

Levee construction and dredged material placement in the tidal marsh area for the selected restoration alternative should compensate for the anticipated settlement. Additional fill material due to settlement is estimated at 10 to 40 percent of the total fill height. The specific techniques used to compensate for anticipated settlement would depend on the findings of the design level geotechnical investigations, but could include: (a) placement of additional fill above the intended finish grade of levees to compensate for anticipated settlement and sea level rise; (b) application of surcharge loads or other settlement acceleration techniques; or (c) avoidance of excessive fill placement.

5.2 Slope Stability

The stability of levees needs to be addressed during the design and construction phases. Side slopes of 3 horizontal to 1 vertical or flatter are used for this conceptual design and evaluation. If dredged material is used for levee construction, it should be determined whether an acceptable margin of safety could be achieved.

5.3 Fill Material Properties

Should it become necessary to use dredged material for levee construction, further investigation is recommended to determine its strength and compaction characteristics. Preliminary data indicate that about 25% by volume of the total imported dredged material could be characterized as cohesionless (i.e. sand), which raises stability concerns. Due to the extremely high moisture content (80% water by volume) of the dredged material, dewatering operation would be required prior to placement and compaction. In addition, during pumping of dredged material to the site, sands and coarser material would settle first followed by fine-grained silts and clays resulting in material segregation. Because of this, it may be necessary to remix the material to ensure even distribution of material from compaction standpoint.

5.4 Seismic Design Considerations

Ground failure hazards such as liquefaction, earthquake-induced settlement, and lurching are processes that involve the displacement of the ground surface resulting from a loss of strength or failure of the underlying materials due to strong seismic ground motions. There is a potential for earthquake-induced lurch cracking to occur at the site during an earthquake (Miller Pacific Engineering Group 1995). A review of seismic hazards, risks and probable ground motions should be done.